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TECHNICAL MEMORANDUM

Date: **November 7, 2008**
To: **Barney Baker, P.E., Baker Design Consultants, Inc.**
From: **Kirk Bosma, P.E., Woods Hole Group, Inc.**
Re: **Prouts Neck Beachfront Protection – Technical Analysis Memorandum**

The Woods Hole Group is pleased to present this technical memorandum evaluating the proposed coastal structures at the Stone and Kohlberg properties in Prouts Neck, Scarborough, Maine. This memorandum presents the results of the technical analysis as proposed in our scope of work (Woods Hole Group, 2008). The technical analysis is geared towards determination of the driving coastal processes at the location, the interaction of the coastal processes with the proposed coastal structures, and evaluation of specific concerns raised by Maine Geological Survey (MGS) and the Maine Department of Environmental Protection. The technical memorandum is divided into five distinct sections as follows:

- A brief summary of existing conditions
- A description of the two proposed conceptual design alternatives that are under consideration
- Details of the technical evaluation of the coastal processes and the interaction with the two proposed alternatives
- A discussion of the results of the technical evaluation in relation to the concerns raised by MGS
- Conclusions

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1. EXISTING CONDITONS

The Stone and Kohlberg properties are located along the southern end of Scarborough Beach, in Scarborough, Maine (Figure 1). The Rockefeller property lies between the two project properties, a beach club lies to the southwest, and additional properties exist to the northeast of the Kohlberg property. Timber bulkheads span from the beach club at the southwestern end of Scarborough Beach and continue to the north throughout the developed portion of the coastline. These timber bulkheads vary in construction detail and age, but generally form a uniform coastal structure intended to protect landward areas from erosion during storm events. The condition of the bulkhead(s) also varies along the beach, with the northern portions of the structure being in better condition than those to the south/southeast. The exception is the beach club bulkhead, which was recently upgraded and rebuilt. In general, the remaining bulkhead(s) along the entire stretch of coastline to the northeast of the beach club are sub-standard relative to current coastal engineering design standards. The beach fronting the structures is primarily median grain sand; however, cobble deposits have formed along the base of a significant portion of the bulkhead(s), particularly along the northern sections.

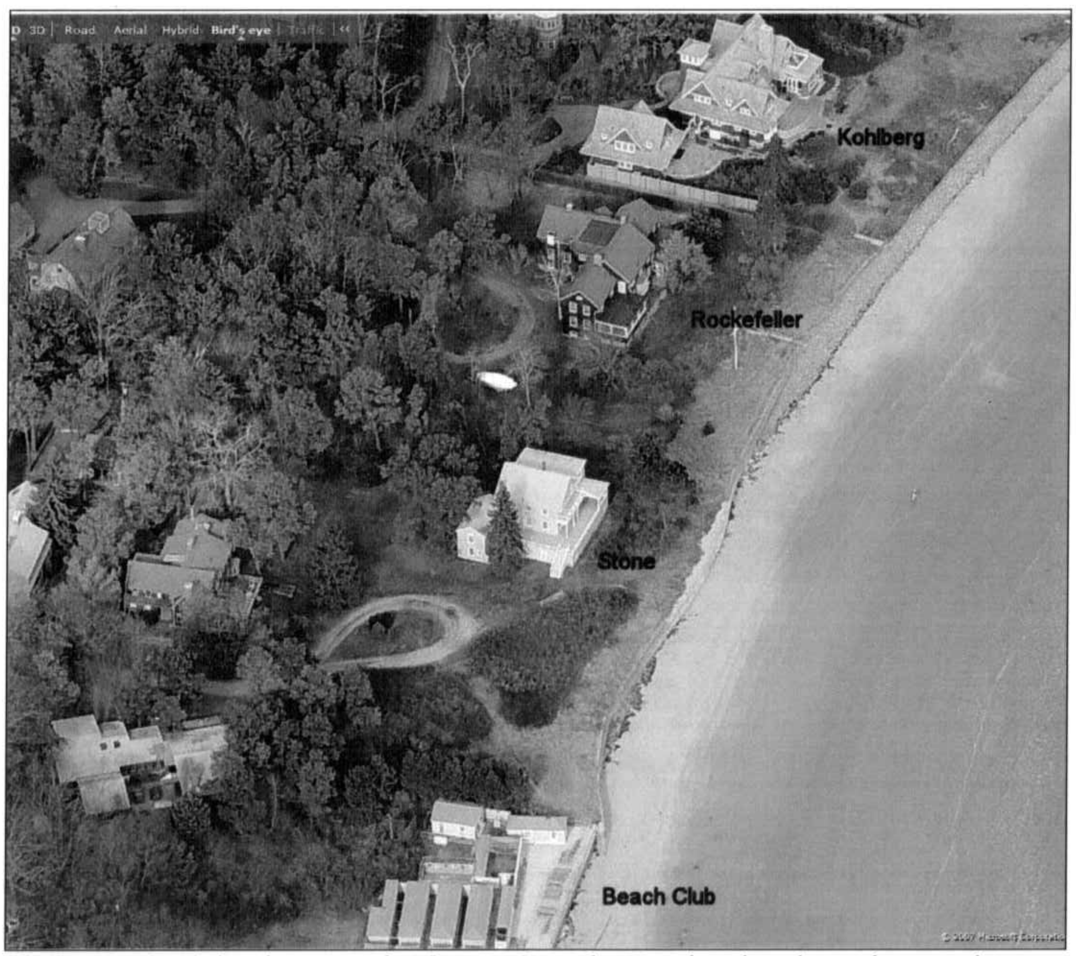

Figure 1. Aerial photograph from Google Earth showing the project and neighboring properties. This aerial was taken prior to the 2007 storm damage.

During the Patriots' Day storm of 2007, the timber bulkhead fronting the Stone property, as well as a portion of the bulkhead fronting the Rockefeller property, failed (Figure 2). This failure resulted in significant erosion of the landward dune region and prompted emergency placement of rip-rap in the erosion area at the Stone property (Figure 2). Although overtopping and overwash occurred along a majority of the Scarborough Beach region, significant damage was limited to this portion of the structure(s). Following the storm event, MGS reported overwash deposits reaching over 150 feet landward of the bulkhead location (MGS, 2008). Prior to the bulkhead failure, the Stone property bulkhead was located slightly seaward of the remaining bulkhead structures, as indicated in Figure 1. This incongruity may have weakened the structure in this region and exposed this section to increased wave energy.

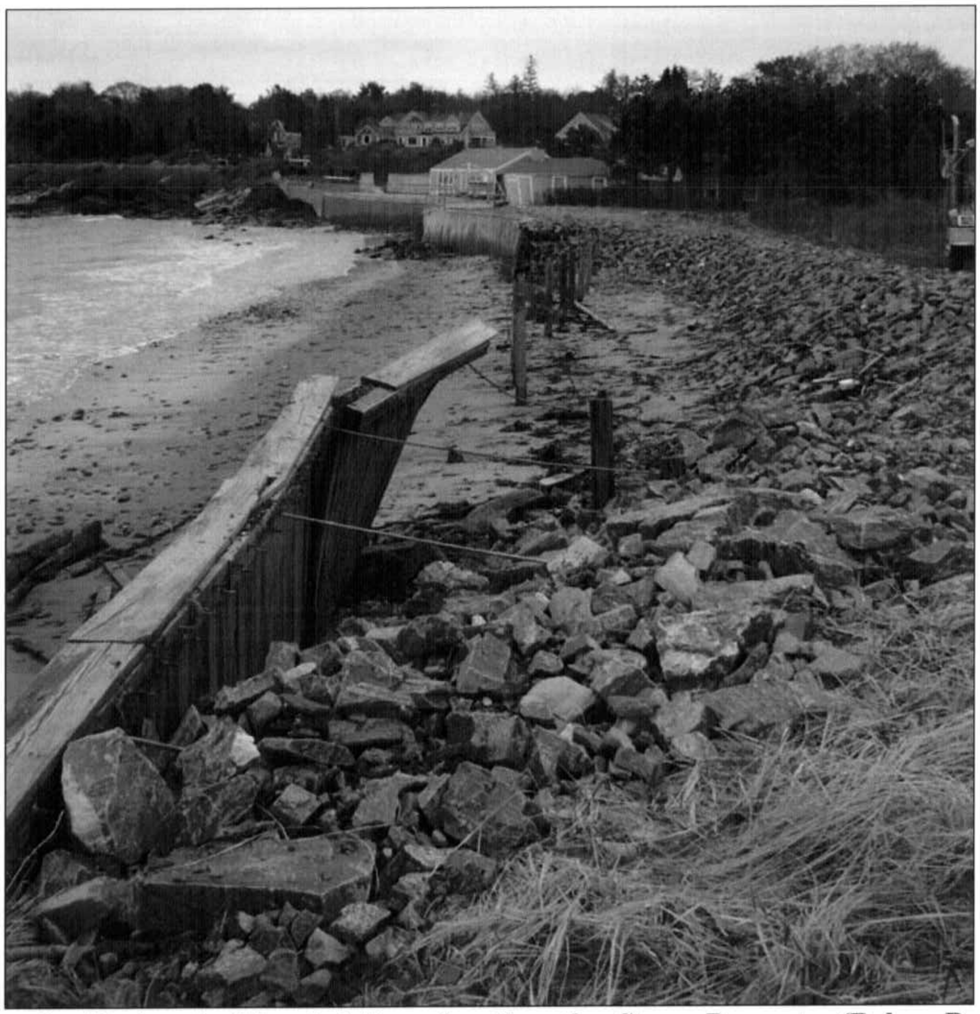

Figure 2. Timber bulkhead failure fronting the Stone Property (Baker Design Consultants, 2008).

Following the bulkhead failure, an application to construct a coastal revetment was presented by Baker Design Consultants for the Stone property. Additionally, the Kohlberg property also sought to replace their timber bulkhead with a stone revetment.

Although the Kohlberg property bulkhead did not experience failure during the Patriots' Day storm of 2007, the bulkhead did experience some damage and is generally in poor to fair condition. As such, the Kohlberg property is also considering replacement of their structure in conjunction with the Stone scope of work. For additional details, a comprehensive summary of the site history, project description, and proposed design can be found in the Application for NRPA Coastal Sand Dune Permit (Baker Design Consultants, 2008), which was submitted to the Division of Land Resource Regulation, Maine Department of Environmental Protection on March 14, 2008.

2. PROPOSED DESIGN ALTERNATIVES

A brief alternative analysis evaluating coastal protection options was presented by Baker Design Consultants (2008) and focused on a vertical bulkhead and stone revetment alternatives. Conceptual designs of these two alternatives were also presented by Baker Design Consultants (2007) and are shown in Figures 3 (new bulkhead) and 4 (revetment). The bulkhead alternative consisted of replacing the historic bulkhead with a new bulkhead structure, as presented in Figure 3. The new bulkhead would consist of 20-foot fiberglass sheet piling, with tiebacks and a stone mattress embedded into the coastal dune. The revetment alternative consisted of replacing the historic bulkhead with a sloping revetment structure, as presented in Figure 4. In section 3 of this letter report, these two alternatives are evaluated in relation to the dominant physical processes at Scarborough Beach to assess the performance and potential impacts of each alternative design.

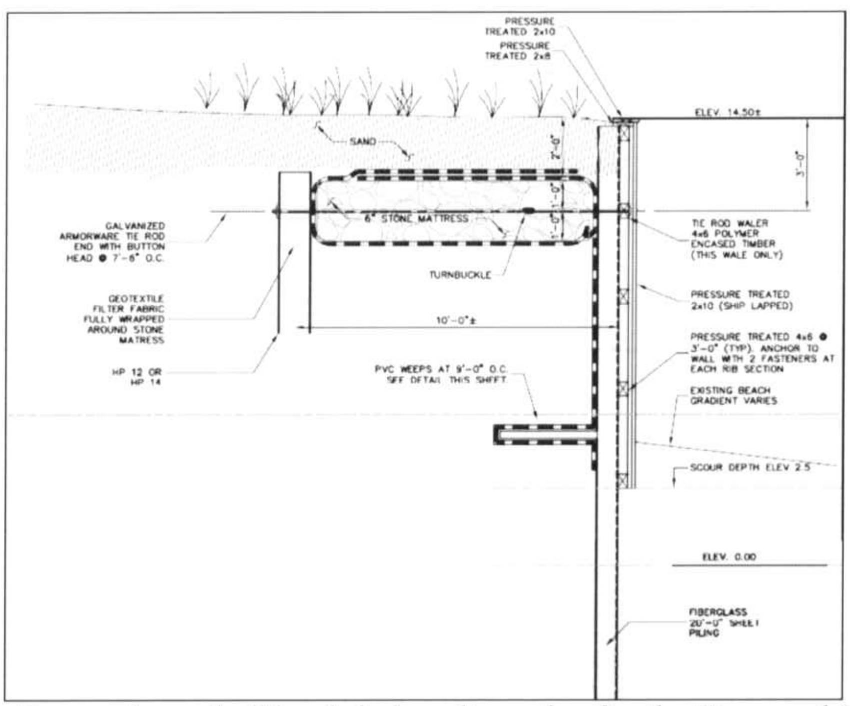

Figure 3. Proposed new bulkhead design alternative for the Stone and Kohlberg properties (Baker Design Consultants, 2007).

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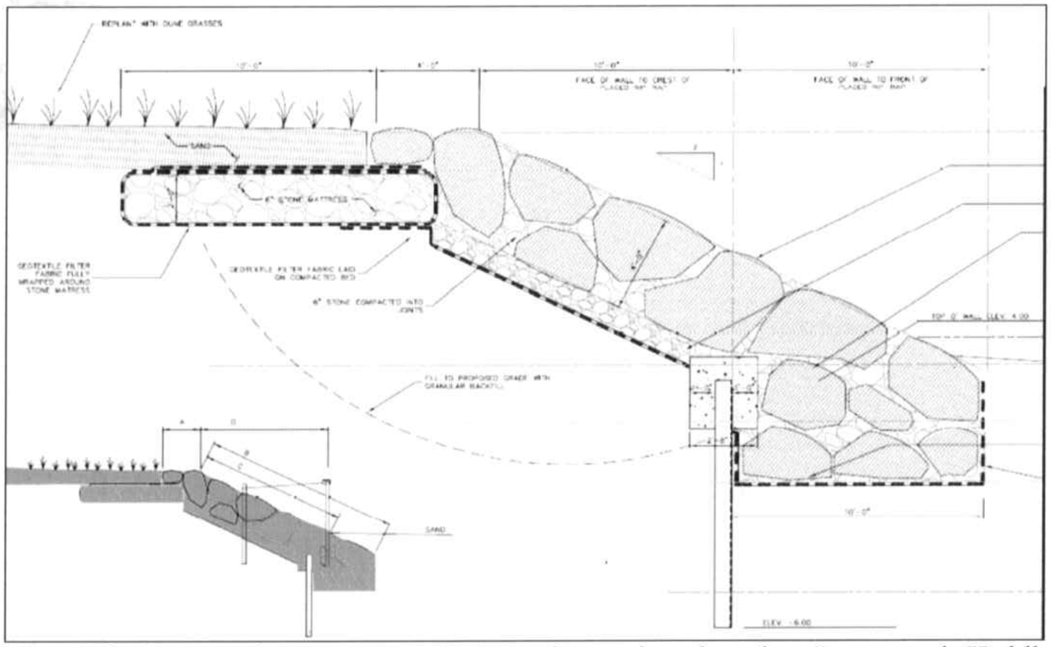

Figure 4. Proposed revetment design alternative for the Stone and Kohlberg properties (Baker Design Consultants, 2007).

3. TECHNICAL EVALUATION

3.1 Water levels, storm surge, and wave conditions

Proper design and evaluation of a coastal structure requires an understanding of the site-specific coastal processes. Therefore, site-specific physical parameters, including design water levels, wave heights and wave periods, were developed for the structural evaluation of proposed coastal structures at the Stone and Kohlberg properties along Scarborough Beach.

A total of 8 design water levels were evaluated, ranging from Mean Lower Low Water (MLLW) to the 1%-annual-chance still water surge elevation (100-year return period) of 9.55 feet. Table 1 presents the design water levels evaluated referenced to the National Geodetic Vertical Datum (NGVD) of 1929. The 1-, 10-, 50-, 100-yr still water storm levels were obtained from the United States Army Corps of Engineers (USACE) Tidal Flood Profiles (1988), while the normal tidal water levels (Mean High Water, Mean Low Water, etc.) were obtained from the National Oceanic and Atmospheric Administration's (NOAA) Center for Operational Oceanographic Products and Services. Baker Design Consultants, Inc. (2008) presented tidal water levels based on the NOAA published data for Portland, Maine. However, the values presented herein more accurately reflect the tidal levels at the Stone and Kohlberg properties near Prouts Neck, which are located approximately 10 miles south of the Portland tidal station.

Additionally, the Federal Emergency Management Agency (FEMA) presents the 100-year flood elevation at 14 feet NGVD 1929 for the region seaward of the existing bulkhead(s). FEMA flood elevation values include not only the still water storm surge elevation, but also the wave height, wave set-up, and wave runup associated with the 100-year flood elevation. The 100-year storm surge elevation presented in Table 1, and provided by the USACE (1988) represents the still water elevation only. In the evaluation sections presented herein, additional physical parameters (e.g., wave height) are added to this still water elevation.

Table 1 also presents the water depth at the structure for each corresponding water level. In order to be conservative, the elevation of the beach is assumed to be in a winter condition, when the beach elevation is approximately 4.8 ft NGVD. Based on observations by both Baker Design Consultants (2008) and MGS (2008), the beach elevation varies between approximately 7.5 feet NGVD in the summer months, and 2-3 feet lower during the winter months (MGS, 2008). As shown in Table 1, the still water level does not reach the bulkhead location under most normal tidal conditions. Therefore, only storm events will interact directly with the proposed structure (whether a bulkhead or a revetment) and a majority of the evaluation focuses on these storm occurrences.

Table 1. Design water levels evaluated at the subject properties.

Water Level	Still water elevation (NGVD 1929, ft)	Water depth at structure (ft)
MLLW	-4.57	N/A
MLW	-4.17	N/A
MHW	4.78	N/A
MHHW	5.42	0.62
1-yr return period	7.19	2.39
10-yr return period	8.58	3.78
50-yr return period	9.25	4.45
100-yr return period	9.55	4.75

Water levels alone do not adequately quantify the physical processes that occur within the coastal zone. For proper design and evaluation of coastal structures, the offshore wave climate, as well as the transformed nearshore waves, needs to be determined. Specifically, storm waves that are associated with the return period water levels are required to assess the proposed coastal structures.

Long-term time series of wave climate are not available for most shorelines, and although there are some ongoing wave data collection efforts within Casco Bay by the Gulf of Maine Ocean Observing System (GoMOOS, 2008), this represents a relatively short time frame of observations (approximately 6.5 years from 2002-2008). For this evaluation, the Wave Information Study (WIS) time series of wave data and wind data were used to describe the wave climate offshore the Prouts Neck region. This data set represents 20-years worth of wave data and can more accurately be utilized to determine return period storm wave heights. WIS, performed by the U.S. Army Corps of Engineers (USACE), has met a critical need for wave information in coastal engineering studies since the 1980s and is widely accepted for design purposes for United States shorelines by many coastal engineers and scientists. WIS contains time series information of spectrally-based, significant wave height, peak period, peak direction, and wind speed and direction produced from a computer hindcast (prediction) model. The hindcast wave model,

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WISWAVE (Resio and Tracy, 1983) is run using wind information (speed and direction) at selected coastal locations around the United States. The model predicts wave climate based on local/regional wind conditions. Wave measurements made by NOAA during the 1980s made verification of the WIS results possible by comparing the statistics and the distributions of wave heights and periods from different time periods (Hubertz et al., 1993). The availability of long-term records makes WIS data attractive when considering average or seasonal wave conditions. Although direct, *in situ* measurements show some difference in detail, the WIS data set provides an accepted and widely used long-term wave data set, which is a significant improvement over representing the sea state with a single wave condition.

The most recent WIS simulations (Phase III-type) were used for this analysis and provide wave parameter results every hour for a twenty-year time period (1980-1999). The Phase III-type WIS data represent the most up-to-date wave generation and wave parameter development. Atlantic Ocean WIS station 37, located in Casco Bay, was used to determine wave conditions offshore of Prouts Neck. The 20-year WIS data set offers a synopsis of the wave climate offshore of Prouts Neck. Figure 5 presents the distribution of significant wave height (illustrated using a wave rose plot) for the WIS station. The grayscale colors indicate the magnitude of the wave height, while the circular axis represents the direction of wave approach (coming from) relative to True North (0 degree). The extending radial lines indicate percent occurrence within each magnitude and directional band. The primary clustering of wave directions tends to be propagating from the southeast, with less frequent, but larger wave events approaching from the northeast.

In order to determine the wave heights associated with return period level storm events, this WIS data were used in an extremal analysis. Extremal analysis represents a methodology that is used to derive a probability distribution from the available data. Then, from the probability analysis, a return period or recurrence interval can be estimated. In general, these techniques are known as "extremal analysis" estimation, and are useful for ordering data that are not deterministic.

The return periods can span various lengths of time. Generally, return values are presented for 1 year, 10 years, 25 years, 50 years, and 100 years, although any arbitrary return period can be calculated. The return period can be thought of as the average period of waiting between events exceeding some specified value. For instance, a 50 year return value of a 5 foot wave height means that for any given year there is a 1/50 chance that a wave of 5 feet will be reached.

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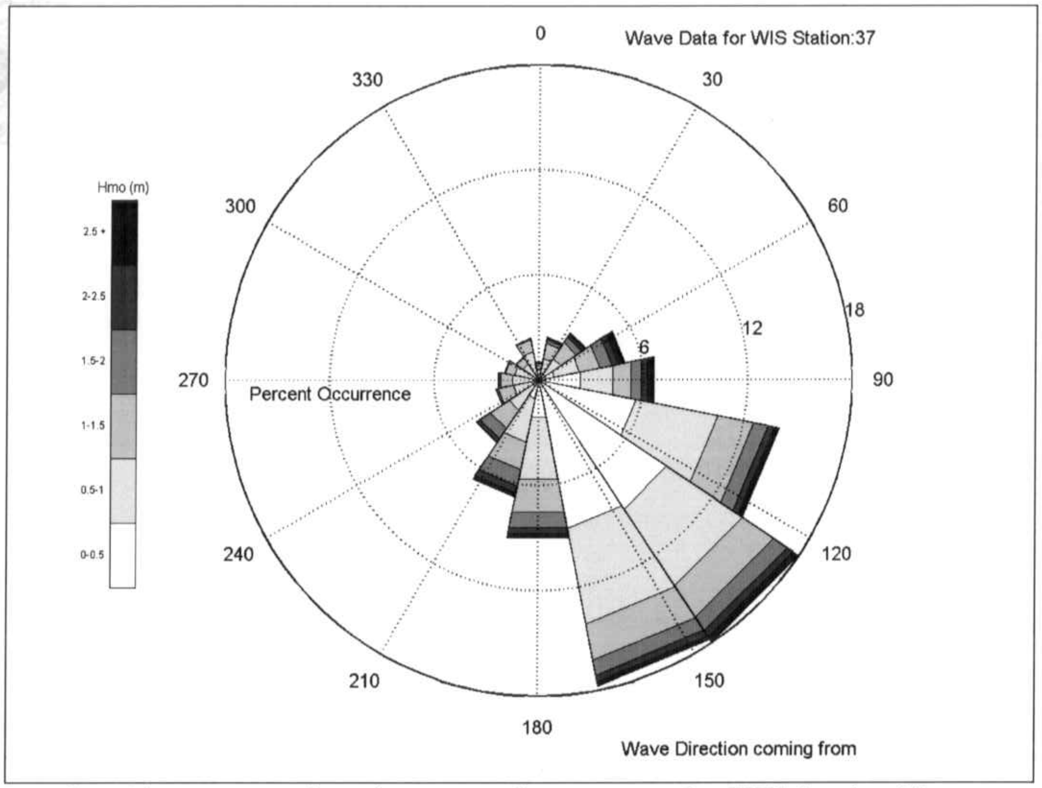

Figure 5. Twenty-year, hourly-averaged wave roses for WIS Station 37.

For this evaluation, the return-period storm wave heights were developed using the Generalized Extreme Value (GEV) method. This method provides reliable estimates of extremes without assuming the distribution type is known (Resio, 1989). The GEV method uses asymptotic methods to fit sampled maxima to the tail of a parent distribution, whose characteristics are estimated from the original sample. The original sample was taken from the WIS 37 station data set. Table 2 presents the offshore return period storm wave parameters determined from the extremal analysis.

Offshore waves propagating to the subject property will undergo significant transformations, including breaking, reformation, diffraction, refraction and non-linear changes. As a wave approaches the shoreline, it will shoal (increase in height). Wave shoaling occurs as the wave enters shallower water. The wave speed and wave length decrease in shallow water; therefore, the wave energy and wave height increase. As the wave continues to heighten, the crest will become too steep and the wave will become unstable and break, dissipating energy in the form of turbulence and work against bottom friction. Although there are more sophisticated methods for transforming a wave into the nearshore zone (e.g., numerical transformation wave modeling), ultimately, the height of the wave is limited by the depth of the water. Therefore, for this evaluation a conservative approach was used. The transformed waves were considered to be the largest wave possible associated with each representative water level (depth-limited) and break directly in front of the structure. McCowan (1894) developed the earliest breaker

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criterion and determined that waves break when their height becomes equal to a fraction of the water depth,

$$H_b \approx \kappa h_b \text{ where } \kappa = 0.78 \text{ and the } b \text{ denotes breaking.}$$

In addition to the offshore storm wave heights, Table 2 also presents the nearshore depth-limited waves used in the analysis for both when the still water storm surge occurs during a mean tide level and when the still water storm surge level occurs in concert with a high tide. Storm events where the peak storm aligns with the peak of high tide represent rare occurrences and should be viewed as the worst-case scenario. When a storm surge temporally aligns with a high tide, the impact can be significant, especially in areas with a large tidal range such as the coastline of Maine. For example, although the Patriot's Day storm of 2007 did not have an extreme storm surge, the storm did align with a spring high tide in the region and heightened the storm damage to much of the Maine coastline. The wave heights presented in Table 2 were used in the assessment of the two proposed structures at the Stone and Kohlberg properties.

Table 2. Return period storm waves offshore (Casco Bay) and in the nearshore vicinity of the structure at the subject properties.

Event	Offshore Wave Height (ft)	Nearshore Wave Height at Structure+ (ft)	Nearshore Wave Height at Structure* (ft)	Peak Wave Period (s)
1-yr storm	11.2	1.9	5.6	9.0
10-yr storm	19.0	3.0	6.9	10.0
50-yr storm	21.3	3.5	7.2	11.0
100-yr storm	22.0	3.7	7.5	11.0

+ = peak storm surge occurs at mean tide
** = peak storm surge occurs at high tide*

3.2 Wave run-up and overtopping

Wave runup and overtopping of the potential structures were determined in order to compare the relative coastal flooding protection afforded by each of the structures. Essentially, wave runup is the measure of how high the water propagates up the structure, while wave overtopping is an estimate of how much water gets behind a structure during a given storm.

Wave runup is defined as the maximum water surface elevation (measured vertically) from the still water level. Figure 6 illustrates the concept of runup and rundown on a simple smooth slope. The runup depends on the height and steepness of the incoming wave, the slope angle, the surface roughness, and the permeability and porosity of the slope. An increase in the permeability of a slope or the roughness of the slope will decrease the level of runup, as water is allowed to inflow into the structure or greater energy is dissipated, respectively. Figure 7 provides an example of the reduction in runup caused by a greater level of permeability.

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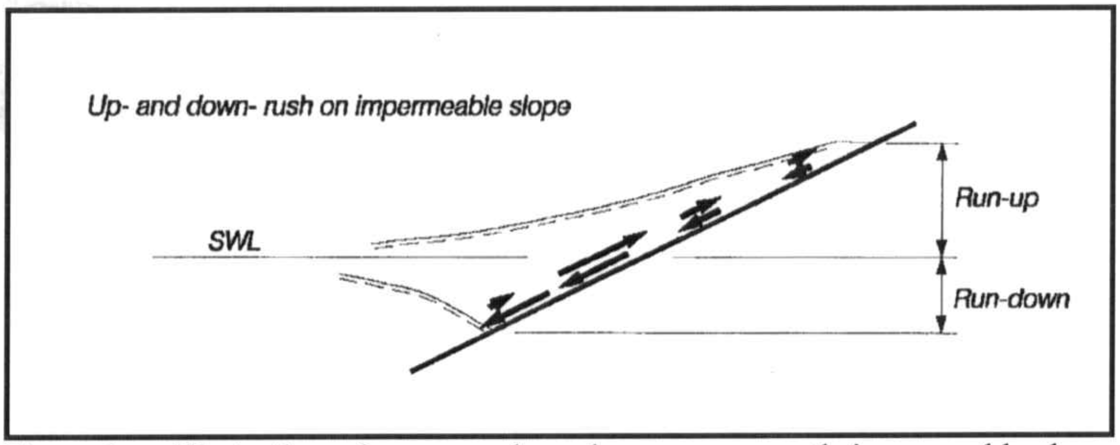

Figure 6. Illustration of runup and rundown on a smooth impermeable slope (USACE, 2006).

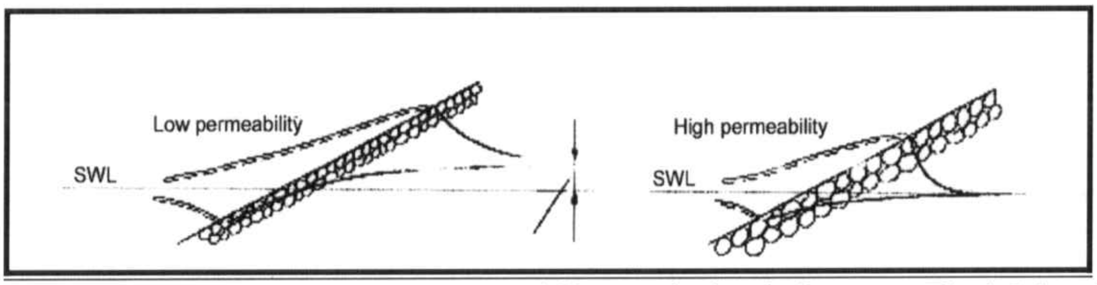

Figure 7. Effect of structures permeability on the level of runup. The left hand panel shows a low permeability structure resulting in increased runup, while the right hand panel shows a high permeability structure with decreased runup (USACE, 2006).

Engineering analysis of the wave runup was conducted using standard engineering methodology for both coastal seawalls/bulkheads and revetments. For the vertical bulkhead, the methodology was developed by Saville (1955, 1956) as presented in the Shore Protection Manual (USACE, 1984). For the proposed rubble sloping revetment, wave runup was calculated using the method of van der Meer and Stam (1992) as:

$$\frac{R_{u2\%}}{H} = 0.835\xi_{op} \qquad \textit{for } 1.15 < \xi_{op} \le 1.72$$

$$\frac{R_{u2\%}}{H} = 1.1\xi_{op}^{0.46} \qquad \textit{for } 1.72 < \xi_{op}$$

where

$$\xi_{op} = \frac{\tan\alpha}{\sqrt{H/L}}$$

with the variables defined as:

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$R_{i2\%}$ is the runup level exceeded by 2 percent of the incident waves,
H is the significant wave height,
α is the slope of the structure
L is the wavelength

Wave overtopping occurs when the highest runup levels exceed the crest of the structure, thereby allowing water to flow over and behind the structure, as shown in Figure 8. Overtopping is presented as a time averaged volume of water that is discharged over the crest of the structure (liters/s-ft). The amount of allowable overtopping depends primarily on the type and function of the particular structure. Wave overtopping is unevenly distributed in time and space and a major portion of the overtopping discharge during a storm is due to a small fraction of the waves. Most estimates of overtopping are based on empirical formula developed in field and laboratory studies. Wave overtopping can result in significant upland flooding or erosion landward of the coastal structure. The ability of a structure to reduce wave runup and overtopping is a key component of the overall shoreline protection. Therefore, a key component of the relative effectiveness of each of the proposed structure is the relative levels of overtopping allowed.

For the proposed vertical structure, the methodology of Franco and Franco (1999) was utilized to predict the overtopping (Q) of the bulkhead.

$$Q = \sqrt{gH^3}\,0.082\exp\left[-3.0\frac{R_c}{H}\frac{1}{\gamma_B\gamma_s}\right]$$

where

H is the significant wave height at the structure
R_c is the freeboard (distance between the structure crest and still water elevation)
γ_s *and* γ_B are geometry and wave crest factors

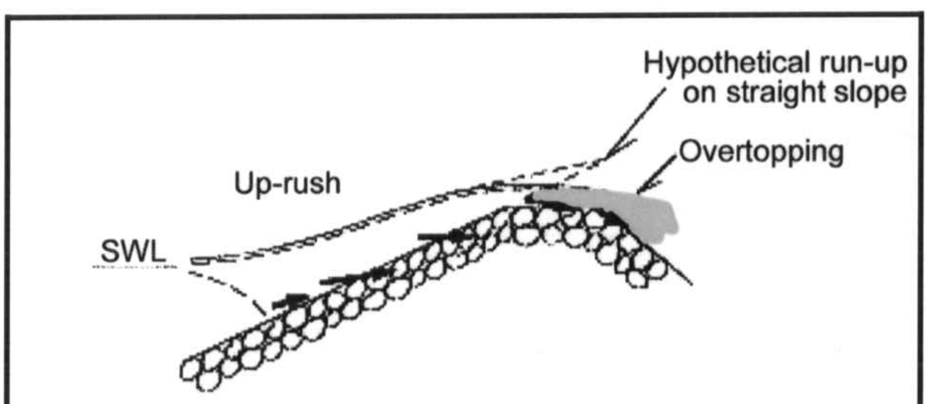

Figure 8. Illustration of wave overtopping of a sloped structure. Overtopping occurs when the runup exceeds the crest of the structure (USACE, 2006).

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For the sloped revetment, the methodology of van der Meer and Janssen (1995) was applied.

$$Q = \sqrt{gH^3}\,0.2\exp\left[-2.6\frac{R_c}{H}\frac{1}{\gamma_b\gamma_r\gamma_h\gamma_\beta}\right]$$

where

H is the significant wave height at the structure
R_c is the freeboard (distance between the structure crest and still water elevation)
$\gamma_r, \gamma_b, \gamma_h$ *and* γ_β are various factors such as the surface roughness, etc.

Table 3 presents the wave runup and overtopping results for both the proposed vertical bulkhead design and sloped revetment design. Due to the low crest elevation for either structure (14.5 ft NGVD); the structures are overtopped during storms of 10-year level or greater. Therefore, limiting the amount of overtopping that occurs during these storm events may provide a protection benefit. Wave overtopping is typically reduced for a sloping rubble structure since the wave energy is more easily dissipated on the face of the structure. The results of the analysis show that for the typical storm events, the amount of overtopping is greater for the bulkhead alternative. For example, during a 50-yr storm event, both alternative structures would be overtopped; however, the rate of overtopping of the revetment would be 0.2 l/s-ft, while the bulkhead would be overtopped at a rate of 0.9 l/s-ft. This represents over 4 times more water spilling into the upland area for a bulkhead versus a revetment. Due to this increased overtopping, the bulkhead would be more prone to damage, as indicated in Table 3.

The results also present storm cases where the peak of the storm surge aligns with a high tide. As discussed, storm events where the peak storm aligns with the peak of high tide represent much rarer occurrences and should be viewed as the worst-case scenario. The estimated probablility of these storms is also presented in Table 3. For example, while the 100-year return period storm represents a 1%-annual-chance of occurrence, the 10-year storm combined with a high tide also reprents approximately a 1%-annual chance of occurrence. When storms combine with a high tide, rapid overtopping of the structure and significant damage to either the bulkhead or revetment design will occur due to the low crest elevation (14.5 ft NGVD) of the proposed structure. When combined with a high tide, storm events of 50-year or greater return period would result in a complete failure of either structure. Ultimately, storms that occur with the perfect matching of peak storm surge and high tide will likely result in significant damage to the upland area independent of alternative selected (unless an increased crest elevation is designed).

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Table 3. Wave runup and overtopping of proposed structures (vertical bulkhead and sloping revetment) at the Stone and Kohlberg properties.

Event	Annual Occurrence Percentage	Proposed Structure	Wave Runup (ft)	Wave Overtopping (l/s-ft)	Level of Structural Damage
1-yr storm	100%	Bulkhead	5.2	0	No damage
	100%	Revetment	6.5	0	No damage
10-yr storm	10%	Bulkhead	Overtopped	0.2	No damage
	10%	Revetment	Overtopped	0.01	No damage
50-yr storm	5%	Bulkhead	Overtopped	0.9	Damage to crest of structure
	5%	Revetment	Overtopped	0.2	No damage
100-yr storm	1%	Bulkhead	Overtopped	1.7	Damage to crest and back dune
	1%	Revetment	Overtopped	0.4	No damage
1-yr storm at High Tide	8%	Bulkhead	Overtopped	45	Damage to crest and back dune
	8%	Revetment	Overtopped	40	Damage to crest if not protected
10-yr storm at High Tide	1%	Bulkhead	Overtopped	*Significant damage*	
	1%	Revetment	Overtopped		
50-yr storm at High Tide	0.4%	Bulkhead	Overtopped	*Failure of structure*	
	0.4%	Revetment	Overtopped		
100-yr storm at High Tide	0.1%	Bulkhead	Overtopped		
	0.1%	Revetment	Overtopped		

3.3 Wave reflection

Waves that impact coastal structures will be reflected off the structure to a certain degree. Reflection of wave energy increases the erosion potential in front of the structure, since the fronting beach would be exposed to both the incoming and reflected wave energy. Reducing the reflection potential of a coastal structure is another key aspect of coastal structure design. The structural parameters affecting wave reflection are the structure slope, the roughness, and the porosity of the structure slope. Increasing the roughness of a structure slope helps to reduce wave reflection because the wave is allowed to flow through a porous surface, which induces breaking and reduces the energy of the wave.

The wave reflection characteristics of a sloping structure can be quantified by the bulk reflection coefficient C_r as derived by Seelig (1983):

$$C_r = \frac{a\xi^2}{(b+\xi^2)}$$

where a and b are coefficients fit by model test results and ξ is the surf similarity parameter given by:

$$\xi = \frac{\tan\alpha}{\sqrt{\dfrac{2\pi H}{gT^2}}}$$

The *a* and *b* coefficient values determined by Allsop (1990), as presented in Table 4, show that increasing the surface roughness of a structure slope reduces the bulk reflection coefficient, and therefore, the reflective characteristics of the structure. This calculation of wave reflection is only valid for cases when the sloping structure is not significantly overtopped. For the proposed Stone and Kohlberg revetment, this represents storm events less than the 50-year return period level. Storm events exceeding the 50-year level will not have significant wave reflection since a majority of the incident wave will transmit over the structure. The methodology presented above was used to assess the reflection from the proposed revetment structure, with coefficients associated with a 1-layer rock structure surface.

Table 4. Wave Reflection Coefficients for Non-Overtopping Sloping Structures (Allsop, 1990).

Author	Structure Surface	*a*	*b*
Allsop (1990)	Smooth	0.96	4.8
	1-layer rock	0.64	7.22
	2-layer rock	0.64	8.85

When the structure is vertical, like the proposed bulkhead at the Stone and Kohlberg properties, the method of Allsop, McBride and Columbo (1994) should be applied, which is given by:

$$C_r = 0.79 + 0.11\frac{R_C}{H_S} \qquad for\ \frac{R_C}{H_S} \le 1.0$$

$$C_r = 0.90 \qquad for\ \frac{R_C}{H_S} > 1.0$$

Table 5 shows the computed bulk reflection coefficients for the proposed revetment and the proposed bulkhead using the two bulk reflection coefficient equations provided. The calculations indicate that when waves impact the structure, the reflected wave energy is significantly reduced for the revetment alternative. In other words, the revetment alternative reduces wave reflection, and subsequently wave energy, by approximately 35% compared to the vertical bulkhead alternative. This has significant impacts on the erosion of the coastal beach fronting each type of structure, which is evaluated in greater detail in the following section (scour).

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Table 5. Computed bulk reflection coefficients.

Proposed Structure	Reflection Coefficient
Bulkhead	0.90
Revetment	0.58

The wave reflection associated with the proposed revetment could be reduced further by implementing a 2-layer design (to a reflection coefficient of approximately 0.56), which is recommended not only for the wave dissipation benefits, but also from a structural stability standpoint.

3.4 Scour, beach impacts, and sand budget

Coastal structures, while limiting upland erosion, can also exacerbate erosion seaward of the structure due to increased wave reflection and energy. The expected wave reflection coefficients for each of the proposed alternatives at the Stone and Kohlberg properties were determined in the previous section. To determine the direct impact on the beach that may be caused by each of the proposed structures, the increased scour and beach erosion is determined in this section. Additionally, a basic sand budget for each alternative is also presented.

Scour caused by breaking wave on vertical structures is generally greater than for nonbreaking waves. It is expected that during storm events, waves will be breaking near or directly at the vertical wall at the Kohlberg and Stone properties. The physical mechanisms for scour are not well understood, but scour is likely caused by strong downward directed flow that occurs at the base of the wall. As a conservative estimate, the scour depth at a vertical wall is approximately equal to the nonbreaking wave height that can be supported by the water depth at the structure (USACE, 2006). More recently, Fowler (1992) conducted a series of experiments to investigate the scour depth in front of a vertical structure. The following empirical equation was determined by Fowler (1992) based on his experiments:

$$\frac{S_m}{H_{mo}} = \sqrt{22.72\frac{h}{L_o} + 0.25}$$

where S_m represents the maximum scour depth and h is the pre-scour water depth at the vertical wall. Fowler's predictive equation does not include any parameters related to sediment properties, although the influence of sediment grain size is likely minor due to the turbulent nature of the scour processes. Table 6 presents the range of scour depths for bulkhead/seawall alternative at the Stone and Kohlberg properties. For this evaluation, scour is expected to occur to the depth indicated in Table 6 at least until significant cobble is encountered, which will reduce the expected scour depth to a certain degree.

Scour at the toe of sloping structures is a function of structure slope, porosity, water depth, grain size, and incident wave conditions. Despite considerable research into wave-induced scour at sloping structures, no empirical estimation method has been proposed.

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Presently, there are no generally accepted techniques for estimating scour at a sloping structure; however, the Coastal Engineering Manual (USACE, 2006) provides some simple rules of thumb for providing engineering guidelines related to the scour at the toe of sloping vertical structures. The maximum scour at a sloping structure is expected to be somewhat less than a vertical wall at the same location and under the same wave conditions. The depth of scour is also expected to decrease with the structures reflection coefficient. Structures with milder slopes and greater porosity will experience less wave-induced scour. Therefore, as a first-order estimate of the depth of scour for the proposed revetment alternative, the scour depth is reduced based on the reflection coefficient calculated in the previous section. Table 6 presents the scour depths expected for the revetment alternative at the Stone and Kohlberg properties.

Table 6. Expected scour depths and beach erosion for the proposed structures (vertical bulkhead and sloping revetment) at the Stone and Kohlberg properties during storm events.

Event	Annual Occurrence Percentage	Proposed Structure	Estimated Scour Depth (ft)	Approximate Eroded Beach Volume (yd^3)
1-yr storm	100%	Bulkhead	1.1-1.9	313
	100%	Revetment	0.6-1.1	100
10-yr storm	10%	Bulkhead	1.8-3.0	800
	10%	Revetment	1.0-1.7	253
50-yr storm	5%	Bulkhead	2.2-3.5	1128
	5%	Revetment	1.3-2.0	378
100-yr storm	1%	Bulkhead	2.4-3.7	1292
	1%	Revetment	1.4-2.1	425
1-yr storm at High Tide	8%	Bulkhead	4.4-5.6	3472
	8%	Revetment	2.6-3.2	1168
10-yr storm at High Tide	1%	Bulkhead	5.4-6.9	5253
	1%	Revetment	3.1-4.0	1750
50-yr storm at High Tide	0.4%	Bulkhead	5.5-7.2	5600
	0.4%	Revetment	3.2-4.2	1901
100-yr storm at High Tide	0.1%	Bulkhead	5.8-7.5	6142
	0.1%	Revetment	3.4-4.4	2112

Table 6 also presents the total expected eroded beach volume occurring during each associated storm event in front of the Stone and Kohlberg properties. This volumetric loss is a first-order estimate of the amount of erosion occurring on the beach due to the wave-induced scour during the storm event. This estimate is likely smaller than the actual erosion that occurs, since actual profile adjustment is a dynamic process that results in a complex shifting of the entire coastal profile. Although more refined erosion volume estimates could be determined using coastal numerical modeling tools, the relative erosion levels presented herein provide the basis for a comparison between the proposed structures.

The bulkhead alternative does result in increased beach erosion when compared to the revetment alternative. For example, during a 50-year storm event, the bulkhead alternative would cause an increase in total erosion of approximately 1,128 cubic yards fronting the Kohlberg and Stone properties. However, the revetment alternative would cause an increase in total erosion of 378 cubic yards, 750 cubic yards less than the bulkhead. The difference between the eroded volumes also increases as the storm level increases. For example, the difference between the eroded volumes during a 50-year storm was 750 cubic yards, while a difference of 867 cubic yards occurs during a 100-year storm.

Although the wave-induced beach erosion is increased by the bulkhead, there will also be a direct loss of coastal dune resources during the construction of the bulkhead or revetment alternative. Therefore, to determine the net impact on the beach and dune environment of each of the two alternatives, a rudimentary sand budget was created. In a traditional sense, a sand budget compares the sediment being added and lost to the beach to determine if a beach is either accreting or eroding. In this simple sediment budget, only the net loss of sand is calculated by evaluating both the immediate loss of material resulting from the construction of a new bulkhead or revetment structure and the long-term loss of sediment caused by the wave-induced scour. As such, the relative impacts of the two alternative structures on the beach can be evaluated.

Construction of a new bulkhead or a revetment at the Stone and Kohlberg properties will result in a direct loss of beach and dune resources. Figure 9 shows the areas of existing dune lost due to the proposed bulkhead, which includes new additions of a stone mattress and anchoring system. Figure 10 shows the areas of dune lost due to the proposed revetment, which includes the structure itself that lies within current beach and dune resource areas, the backsplash mattress, and the area of removed dune that currently would reside behind the crest of the bulkhead. The disturbed and replanted areas are not included in this volume calculation since they are expected to be approximately the same in either alternative. Additionally the calculation does not consider that a majority of the Stone property has already been eroded. For the bulkhead alternative, the total direct construction loss of dune area is approximately 300 cubic yards (175 cubic yards for the Stone property and 125 cubic yards for the Kohlberg property). For the revetment alternative, the total direct construction loss of dune and beach area is approximately 2,340 cubic yards (1435 cubic yards for the Stone property and 905 cubic yards for the Kohlberg property).

Table 7 presents the net loss of sediment, which includes the direct construction loss and the wave-induced scour loss for each storm scenario. In the short-term, the direct construction impact to the dune results in a more net material lost for the revetment alternative, due to the increased horizontal footprint of the structure. However, over the long-term with either increased storm quantity and/or magnitude, the net loss of material is significantly larger for the bulkhead alternative. For example, following the occurrence of a 10-yr storm and a 1-yr storm at high tide, the revetment alternative would have 811 cubic yards less erosion than the bulkhead alternative. Ultimately, the long-term health of the local beach system is better served by the revetment than the bulkhead, as the vertical structure would increase erosion rates on the beach during storm events.

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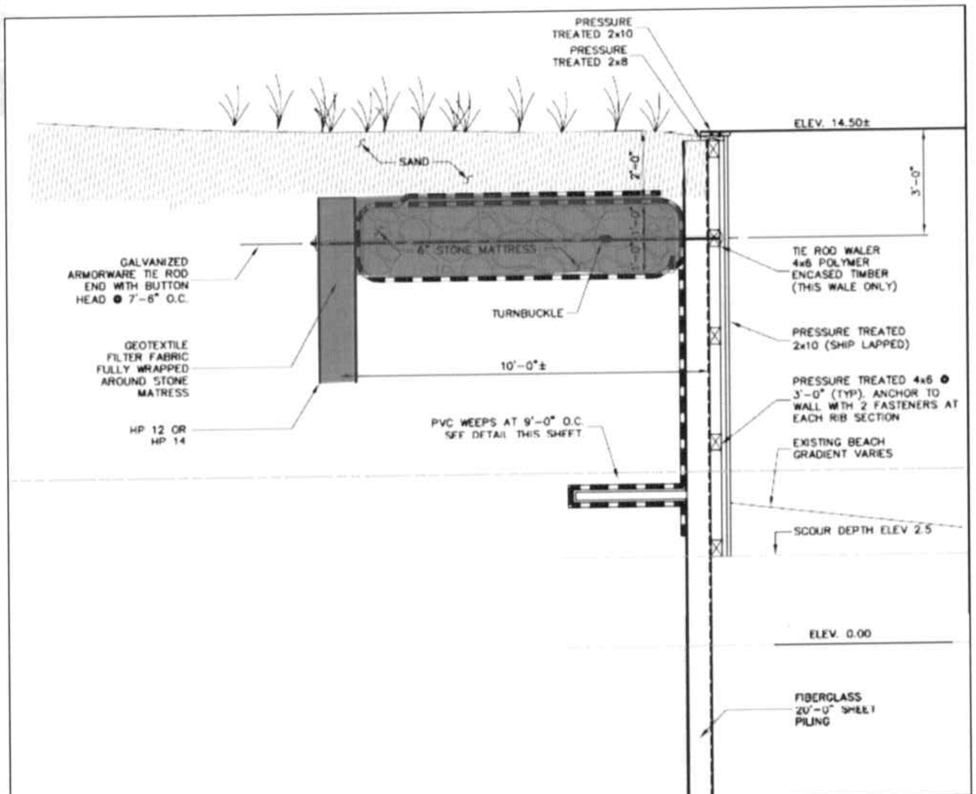

Figure 9. Dune area lost due to proposed bulkhead alternative at the Stone and Kohlberg properties.

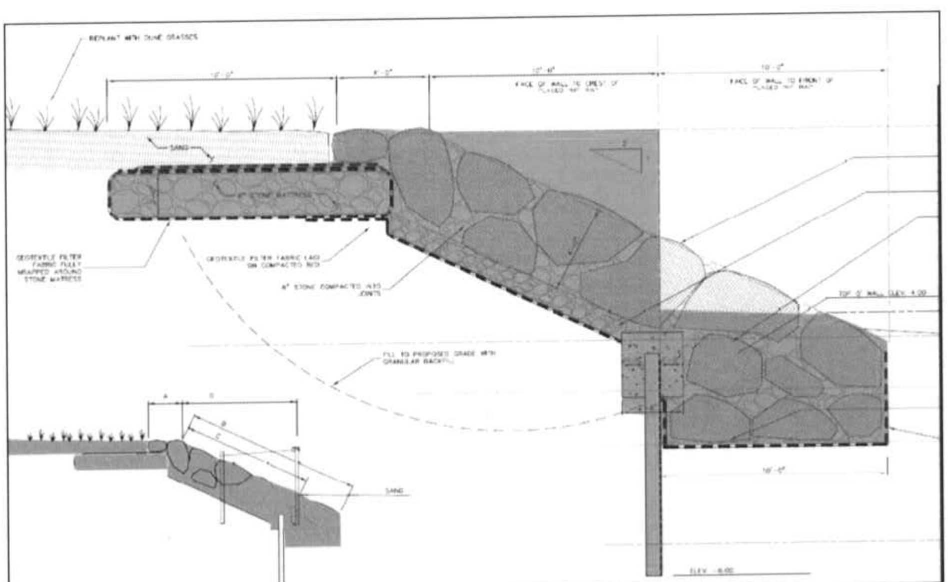

Figure 10. Dune and beach area lost due to proposed revetment alternative at the Stone and Kohlberg properties.

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The initial loss of material from the system can also be mitigated by adding material to the system. If the revetment alternative is preferred, Woods Hole Group recommends additional material is placed to mitigate the initial construction loss. Material should be added to enhance the dune resources landward of the structure, as well as to add beach cover to the toe of the revetment. It is also recommended that any clean, beach compatible material that is removed from the dune be reused to enhance the local resources.

Table 7. Net loss of sediment due to direct construction and wave-induced scour for the bulkhead and revetment alternatives.

Event	Annual Occurrence Percentage	Proposed Structure	Direct Construction Loss (yd^3)	Net Loss of Beach Volume (yd^3)
1-yr storm	100%	Bulkhead	300	613
	100%	Revetment	2340	2440
10-yr storm	10%	Bulkhead	300	1100
	10%	Revetment	2340	2593
50-yr storm	5%	Bulkhead	300	1428
	5%	Revetment	2340	2718
100-yr storm	1%	Bulkhead	300	1592
	1%	Revetment	2340	2765
1-yr storm at High Tide	8%	Bulkhead	300	3772
	8%	Revetment	2340	3508
10-yr storm at High Tide	1%	Bulkhead	300	5553
	1%	Revetment	2340	4090
50-yr storm at High Tide	0.4%	Bulkhead	300	5900
	0.4%	Revetment	2340	4241
100-yr storm at High Tide	0.1%	Bulkhead	300	6442
	0.1%	Revetment	2340	4452

3.5 Structural transitions

Structural transitions are important components for both the engineering design of the structure, as well as for the impacts to the surrounding beach environment. The orientation of bends, corners, and transition sections relative to the incident waves may result in changes to the local wave characteristics, including wave reflection and beach scour. Changes in the waves impacting the structure can also affect structural stability of the corner itself and the entire structure. The bulkhead alternative does not create any transitional areas, curves, or bends since the adjacent bulkheads could tie directly into the proposed bulkhead. The revetment alternative; however, will require 4 transitional areas, including:

- A transition between the beach club seawall and the Stone revetment
- A transition between the Stone revetment and the Rockefeller bulkhead
- A transition between the Rockefeller bulkhead and the Kohlberg revetment
- A transition between the Kohlberg revetment and the properties to the northeast

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Baker Design Consultants, Inc. (2008) presents the proposed transitional designs as abutments in design sheet C-5. These abutments create a concave corner between the proposed revetment alternative and the existing bulkhead structures. Goda (1985) provides a simple formula for estimating the increased wave height at the apex of a concave corner formed by two impermeable walls as:

$$\frac{H_c}{H_i} = \frac{2\pi}{\beta}$$

where H_c is the wave height in the corner, H_i is the incident wave height, and β is the angle formed by the concave corner. The formulation presented for Goda (1985) is for vertical walls and not sloping structures. For steeply sloping structures, this formulation can be applied as a conservative estimate of the increased wave energy caused by the transitional regions. Generally, milder structure slopes, increased porosity, and reduced corner angle will all contribute to reduced wave energy in the concave corner region. As such, the estimated energy increase in the transitional corner is a conservative estimate in the revetment design.

The proposed abutments only interact with the incident waves during storm events. During normal tidal and wave conditions, the abutments will have minimal impact on the structure of the beach. However; based on the currently proposed design for the transitional abutments, Goda's equation indicates that the wave height at the apex of the abutment could be up to 8 times greater than the wave height at the rest of the structure during a storm. Although this level of increase is not feasible based on the local water depth, the analysis does indicate that significant energy increase would occur near these abutment regions and could result in increased runup and overtopping in these regions. These are localized impacts that would not significantly impact the rest of the structure or the beach itself; however, in the transitional region itself, this could lead to increased erosion and structural instability. Therefore, Woods Hole Group recommends the following:

(1) Although the current abutment design is structurally robust, consideration should be given to redesigning these transitional areas to reduce the angle of the transition, improve wave energy dissipation, and bolster protection of the neighboring properties. Specific design recommendations can be provided if the revetment alternative is the final preferred solution.

(2) The number of transitions should be reduced by inclusion of the Rockefeller property, if feasible. There are significant benefits, both structurally and environmentally, to having a contiguous structure spanning the coastline. This would reduce the number of transitions to two and improve the overall performance of the revetment and the beach.

4. DISCUSSION OF RESULTS RELATIVE TO MGS CONCERNS

Based on the technical analysis presented in Section 3, this section provides a direct response to the primary technical concerns raised by MGS (2008).

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MGS Comment: *We recommend that a dune crest at both locations be constructed landward of any proposed seawall so that the dune crest elevation exceeds the published V-zone BFE.*

- Woods Hole Group agrees that an increased dune elevation landward of any proposed coastal structure should be considered as a component of this project.

MGS Comment: *The bulge in front of the Stone property that existed prior to the washout should not be replicated in the proposed seawall construction.*

- Woods Hole Group agrees that this incongruity should be removed from any proposed structure, whether a revetment or seawall. The current design plans (Baker Design Consultants, 2008) have removed this bump in the planform design.

MGS Comment: *The proposed project includes the use of granite abutments (Sheet C-5) to transition back to vertical seawalls at the ends of both projects. We are concerned about end-effect erosion that may occur at these abutment locations, as the transition between a vertical feature and a sloped feature (such as the seawall proposed) can lead to focused wave attack. This could lead to a washout of the adjacent dunes, including the Rockefeller property located to the north of the Stone property.*

- Although these abutments do not have a regional impact, Woods Hole Group agrees that the current abutment design creates significant wave energy increases in the local transitional areas (section 3.5). Therefore, Woods Hole Group recommends that the design of these transitional areas be reconsidered if a sloped revetment structure is proposed. Although the current design is structurally robust, during storm events, there will be undue increased wave energy at these concave areas. Additionally, in order to reduce the overall number of transitional areas, the design approach should encourage the incorporation of the Rockefeller property, if feasible.

MGS Comment: *As a whole, we are concerned about the impacts of the increased horizontal footprint of a sloped wall at the proposed locations, and are not sure whether or not the decrease in wave reflectivity (no calculations have been provided, but we assume reflectivity of a sloped structure to be less than that of a vertical structure) afforded by the sloped wall would outweigh the negative impact of the increased footprint of the seawall.*

According to the submitted plans (Sheet C-6), the proposed project at the Stone property would result in the loss of approximately 3,843 SF of frontal dune. This number was attained by combining the area of loss designated as D (behind former seawall to top of new sloped revetment wall), and area A (top of new wall to replanted grass). At the Kohlberg property, the existing frontal dune loss would be approximately 2,365 SF, for a total of 6,208 SF (0.14 Ac.) of frontal dune that is lost as part of the proposed project.

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- Sections 3.2 through 3.4 provide a comprehensive comparison of the wave overtopping, wave reflectivity, scour, and potential impact to beach resources of the bulkhead and revetment alternatives. The analysis indicates that a sloping revetment structure reduces overtopping, reduces reflected wave energy by approximately 35%, significantly reduces scour of the beach fronting the structure, and in the long-term provides improved beach performance. Although there is an initial loss of dune resources due to the expanded horizontal footprint, the energy reduction and performance gains do indicate the relative long-term benefits of a sloped structure compared to a vertical structure. However, due to the immediate impact to the coastal dune resource, the design should also consider enhancing the landward coastal dune resources (dune enhancement), as well as the seaward coastal beach resources (beach nourishment) during project construction. In other words, additional beach compatible material should be delivered to the project site to mitigate the immediate loss of dune resource area.

  Section 3.4 also provides an assessment of the volumetric impacts to the coastal resources, which provides more detail than frontal dune area alone. This basic sediment budget calculates the immediate short-term construction impacts, as well as the wave-induced scour impacts after construction.

MGS Comment: *The seaward toe of the proposed rip-rap slope extends about 7.5 feet seaward of the "typical line of existing vertical seawall" (Sheet C-6 in both applications). Consequently the proposed project would extend farther seaward than the wooden bulkhead seawall and thus take up space on the beach. In our opinion, this incursion on the beach – seaward of the pre-storm footprint of the former bulkhead seawall - would unreasonably interfere with seasonal beach sediment movement.*

- Woods Hole Group understands that Baker Design Consultants designed the seaward extension of the proposed revetment to limit the increased offset at the transitional areas, as well as to decrease the immediate impact on dune resources. Therefore, the minor seaward advance must be weighed against the potential transitional impacts and the volume of direct dune loss. From a coastal processes basis, the minimal seaward extension of the proposed revetment seaward will not significantly impact the littoral processes that occur during normal wave and tidal conditions since the structure is located at the upper end of the wave uprush zone. During storm events, both proposed structures will be exposed to significant storm waves and the improved energy dissipation of the revetment may outweigh the impact of the minimal seaward extension. Ultimately, this seaward encroachment is not expected to have significant impacts to the natural, alongshore sediment transport patterns. The structure could be moved further landward, as long as additional dune resource loss is not significant.

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MGS Comment: *From a functionality standpoint, we do not expect the two small, sloped walls that would result from the proposed projects to have a large positive impact on the beach seaward of the walls. The proposed projects would result in two small, independent stretches of seawall that would be sloped rip-rap, with the remaining sections of the wall vertical in nature. As a result, it is possible that wave runup and overwash during storm events may actually be worse at these two locations due to wave focusing. This is a detriment to the habitable structures, but possibly would benefit to the natural functionality (i.e., overwash and sediment deposition) of the backside of the frontal dune and the back dune system.*

- Although a more contiguous revetment would have an even greater positive impact on the health of the beach, the Stone and Kohlberg sloped structures would result in a noticeable improvement in the reduction of overtopping rates and beach erosion during storm events. Sections 3.2 through 3.4 quantify the level of improvement when using a revetment versus a bulkhead at these two sites specifically. It has been our experience that even in small local settings, areas protected by a revetment next to areas protected by a seawall have shown reduced beach scour and erosion. Wave runup, overtopping, and scour will not increase at these two locations due to wave focusing or any unnatural process caused by the structures.

MGS Comment: *Because the frontal dune in this area is natural dune vegetation, impacts of the sloped wall design on the existing frontal dune would be greater than if the area landward of the existing seawalls at both properties were developed with impervious surfaces (such as sidewalks and roads) or other development as defined in the Coastal Sand Dune Rules. If many contiguous properties were involved in the project, the potential for reducing reflectivity and improving beach function may be realized, however the sediment exchange between the dune and beach is even less likely to occur in the future. In the cases reviewed here, we do feel that the negative impacts to the frontal dune (physical removal of existing dune and vegetation and replacement with sloped rip-rap walls) outweigh the benefits of a proposed sloped wall construction.*

- It is unclear how impervious surfaces would offer a reduced impact than a coastal structure when considering the function of a coastal dune as a sediment supply to the coastal beach. The sediment exchanged between the dune and the beach will be limited by either a new bulkhead design of coastal revetment, as the intent is to limit upland erosion of the dune. However, due to the low crested nature of both the revetment and bulkhead alternatives, it is expected that the amount of material exchanged during significant storm event will be approximately the same. Both structures are overtopped in events that exceed the 1-year return period level, and as such have opportunity to provide sediment to the beach during storm events. In addition, during catastrophic storm events, either alternative structure fails and will result in erosion of the frontal dune region.

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5. CONCLUSIONS

- Overall, this technical memorandum has quantified the structural interaction with the coastal processes at the Stone and Kohlberg properties. Comparisons were made between two structural alternatives, a vertical bulkhead and a sloping revetment. This quantification should provide the ability to more accurately consider the relative merits of sloping structure versus a vertical structure, specifically related to the long-term benefits and the short-term direct loss of coastal dune resource area.

- Wave overtopping rates are reduced for the revetment alternative when compared to the bulkhead alternative at the Stone and Kohlberg properties. This may result in reduced upland erosion as well as improved structural stability.

- Reflected wave energy is reduced by approximately 35% for the revetment alternative when compared to the bulkhead alternative.

- The revetment alternative has reduced scour depths, as well as reduced volumetric erosion of the beach. In the short-term, the direct construction impact to the dune results in a more net material lost for the revetment alternative, as the due to the increased horizontal footprint of the structure. However, over the long-term with either increased storm quantity and/or magnitude, the net loss of material is significantly larger for the bulkhead alternative.

- If the revetment alternative is preferred, Woods Hole Group recommends additional material is placed to mitigate the initial construction loss. Material should be added to enhance the dune resources landward of the structure, as well as to add beach cover to the toe of the revetment. It is also recommended that any clean, beach compatible material that is removed from the dune be reused to enhance the local resources.

- As currently designed, the transitional areas will produce increased wave energy and may result in increased overtopping and erosion (both the local beach area and the upland dune resources). Therefore, consideration should be given to redesigning these transitional areas to reduce the angle of the transition, improve wave energy dissipation, and bolster protection of the neighboring properties. Additionally, in order to reduce the overall number of transitional areas, the design approach should encourage the incorporation of the Rockefeller property, if feasible.

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